CHECKING ROBUSTNESS FOR A CONCRETE ROADWAY VIADUCT

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Abstract: Experts' pursuits in constructions are connected now with the structural robustness' notions and how to prevent the partial, full or disproportioned collapse. The question: "What is robustness?" has by far no unitary and simple answer. For bridge's structures there are no specific directives for robustness' calculation. Even in COST – Robustness of Structures – TU0601 experts' discussions, bridges problem was less exemplified because of the every structure uniqueness. The object of this study is a concrete roadway viaduct, with eight spans, for two traffic lanes. This paper focuses on two questions: (i) How the use of a type or another of finite element will influence the initial damage's modelling. (ii) The effect of the initial failure type for the inherent dynamic response of the structure and, actually, what element for that answer – the value of the inherent period or the form of the eigenvalue – is more susceptible for the initial type of failure.

Keywords: bridge, initial damage, robustness, structural modelling.

1. INTRODUCTION

A quality design of a building implies taking in care more than the minimized design requirements from a technical regulation.

The current method for the study of robustness of a structure is to propose scenarios in which they are removed (due to various reasons) structural elements. It aims at finding technical solutions to ensure that the structure maintains its ability to redistribute efforts and the functionality in the period right after the removal of the structural elements. This approach is required by that society which, expending resources, expects the engineers to create constructions that maintain functionality even in outstanding circumstances.

In the papers of some specialists (Faber *et al.* [8], Maes *et al.* [14]) is shown that the engineers must act having in the mind that:

$$\text{Cost of robustness measures} \leq \text{Reduction of failure consequences.}$$

The introduction from the design phase of a study on robustness has at the moment at least two grounds: (i) the terrorist attacks which have now a growing probability as a possible action and (ii) the development of the past three decades

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on the technology of materials, nonlinear analysis technique, etc., resulting in the design of structures more economically but also with a lesser margin of safety to unforeseen actions.

2. ABOUT ROBUSTNESS

Experts' pursuits in constructions are connected now with the structural robustness' notions and how to prevent the partial, full or disproportioned collapse. The answer to the simple question "What is robustness?" is far from simple.

Civil engineers have their own approach. Many definitions and mathematical formulas attempt to present the complex phenomenon that is robustness.

Some of the current definitions the experts propose for the concept of "robustness" are presented:

* In Eurocode "Robustness is the ability of a structure to withstand events like fire, explosions, impact or the consequences of human error, without being damaged to an extent disproportionate to the original cause" [Eurocode EN 1991-1-7:2006].

* In COST-TU0601 papers [24] "A structure shall incorporate robustness through consideration of the effects of all hazards and their probabilities of occurrence, to ensure that consequent damage is not disproportionate to the cause. Damage from an event with a reasonable likelihood of occurrence shall not lead to complete loss of integrity of the structure. In such cases the structural integrity in the damaged state shall be sufficient to allow a process system close down and/or a safe evacuation".

* In the Joint Committee on Structural Safety (JCSS) Probabilistic Model Code [23], the following robustness requirement was formulated: "A structure shall not be damaged by events like fire, explosions or consequences of human errors, deterioration effects, etc. to an extent disproportionate to the severeness of the triggering event".

* In professor Uwe Starossek's conception [15], robustness is a desirable property of structural systems which mitigates their susceptibility to progressive disproportionate collapse. It is a property of the structure alone and independent of the possible causes and probabilities of the initial local failure. To define the robustness, Starossek and Haberland [18] proposed the schema in Fig. 1.

Some of the most complete recommendations for buildings are provided by the U.S. General Services Administration (GSA) [22]: *Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Projects*, dating from June 2003.

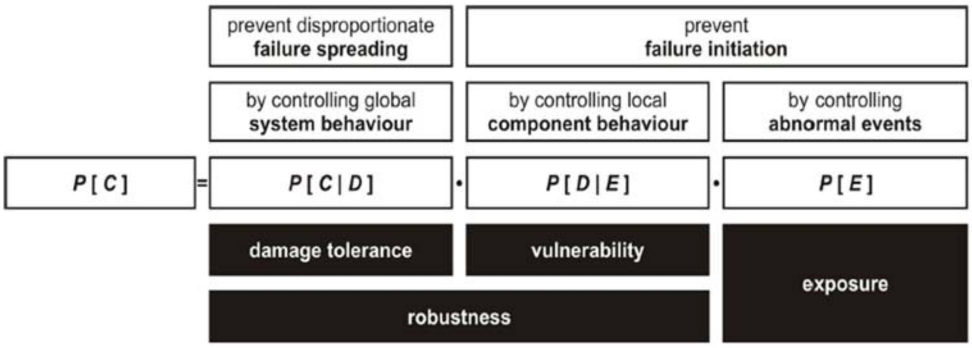


Fig. 1 – Schema defining robustness (Starossek and Haberland [18]).

For bridges there are no specific indications concerning the calculation of robustness. Even in discussions of specialists from COST – Robustness of Structures – TU0601, the bridges problem was less exemplified because of the uniqueness each structure represents. Though, professor Diamantidis [6, 7] shows that: “The requirement to avoid progressive collapse in case of local failure is an important design criterion for multi-span bridges.”

This failure could be type of the effect of a ship impact, strong ice formations collision on a pier, fire, explosion or a major earthquake.

The difference between the progressive collapse and the global one is given by two characteristics:

- its triggering by the failure of a relatively limited area;
- the existence of a time lag until the total collapse.

Below are presented two of the best known examples of progressive collapse due to earthquake: Cypress Viaduct - Loma Prieta 1989 (Fig. 2); Chile 2010 (Fig. 3).

Fig. 2 – Cypress Viaduct - Loma Prieta earthquake, 1989.

Fig. 3 – Chile earthquake, 2010.

Recent disasters and new considerations on risk theory indicate that technical regulations should be improved to more clearly recommendations on this problem.

As a result, my proposal is that, through a broad and encompassing study, to answer the following questions:

1. How employing finite elements of one type or another affects modeling the initial failure;
2. The effect of the initial failure type over the structure's dynamic response – the value or the shape of the eigenvalue is more sensitive to the type of initial failure;
3. If from the previous study it can be discerned when an additional study is needed on the robustness of the structure;
4. If a structure well composed form the seismic response point of view is sufficiently robust for any other extreme loads – explosions, shock, etc.

This paper focuses on the first two questions.

3. CASE STUDY

Description of the structure

The object of this study is a concrete roadway viaduct, with eight spans, for two traffic lanes [2, 5]. The superstructure is made up by precast prestressed section girders of 33.00 m length and 1.80 m height, simply supported, namely four items in cross section. The substructure consists in seven piers (of different height – P1=20.10 m, P2=31.90 m, P3=31.90 m, P4=37.50 m, P5=16.10 m, P6=14.10 m, P7=10.30 m) and two abutments, as shown in Fig. 4. The foundations are solid and spread. Two hinged bearings are placed on piers P1, P5 and P7, guided bearings on piers P4 and P6 and one hinged plus one guided bearing are placed on piers P2 and P3.

Having in view the height of the piers and the way the girders rest on them, let us choose a section made out of piers P2, P3, P4 and the superstructure corresponding to the two spans (see Fig. 4).

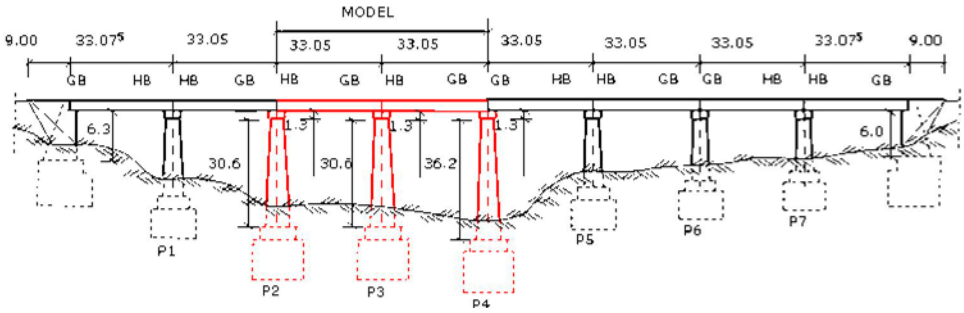
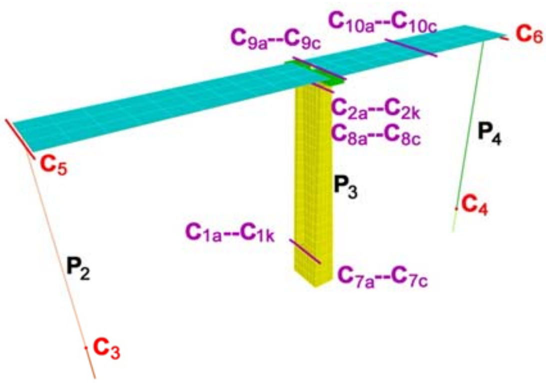


Fig. 4 – Roadway viaduct.

Calculation model

The calculation model is a three-dimensional one, as shown in Fig. 5. The piers have lamellar elevations with H section on their upper part and right-angled section at their bottom lower part. Pile 3 has 31.90 m total height of which 30.60 m is the lamellar elevation (with H section on 25.60 m and right-angled section on 5.00 m) and 1.30, the cross head.

This pier was modelled using three-dimensional finite elements with dimensions chosen so that to follow closely the shape of the pier. The Pier 2 and Pier 4 (the highest) as well as the superstructure girders are modelled with one-dimensional elements of equivalent section. The over-concreting plate is modelled using bi-dimensional elements.



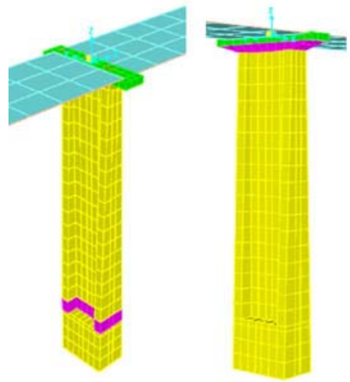


Fig. 5 – Calculation model. Fig. 6 – Modelling the hinges at the central pier.

The bearings of the girders are distributed on the three piers in the following way: the hinged bearings on the pier P2; on the pier P3 the girders from the pier P2 have plain bearings and those coming from the pier P4 have hinged bearings; on the pier P4 there are plane bearings. The bearings are out of neoprene. They are modelled in such a way either to stop or to allow the displacements (translation and rotation) corresponding to the type and characteristics of the bearing.

A special attention was given to loads. To study the specific dynamic characteristics the masses are obtained from the combination [1 × (specific weight of the resistance structure and of the non structural elements) + 0.2 × (service loads)].

Study scenarios

The scenarios proposed to remove some structural elements are divided into two categories: (i) initial damages at the piers level and (ii) initial damages at the superstructure level.

The study aims to determine the way in which the modelling of the same finite structural elements may also impose different approaches of the introduction of the initial damages into the model. For the piers damages, I have proposed the following: (i) for the three piers, to transform some sections into hinges (above the right-angled area and at the level of the cross head, in Fig. 6); (ii) for the pier P3 to remove part of the section. For the damages at the superstructure level scenarios are proposed with different degrees of damages in the supporting area and at the section in the middle of the span.

If a damage of the type when a section of the pier changes into a hinge is taken into consideration, now the question is how to model this hinge at the level of pier P3, which was modelled with three-dimensional finite elements. For the lateral piers – P2 and P4 – which are modelled with one-dimensional finite elements, this scenario was easy to be modelled by introducing hinges into the existing joints, in the calculation model. In order to model the hinges into the pier sections with three-dimensional elements, it is proposed to introduce a new material with a different longitudinal elasticity module (Fig. 6). The variation of the longitudinal elasticity module was chosen to range between $E:10^6$ and $E:4/3$; 11 cases are proposed with the values given in Table 1. The value of the specific weight of the material in these areas is 2.5 kN/m^3 for all cases.

The entire study is based on 38 scenarios of the removing of some elements, in the following way:

C1_low – hinge at the lower part of the pier with three-dimensional finite elements (P3) with the 11 values cases for the longitudinal elasticity module: (a) soft; (b...h) medium; (i...k) strong;

C2_upper – hinge at the upper part of the pier with three-dimensional finite elements (P3) with the 11 values cases for the longitudinal elasticity module: (a) soft; (b...h) medium; (i...k) strong;

C3_hinge at the lower part of the left pier (P2);

C4_ hinge at the lower part of the right pier (P4);

C5_ hinge at the upper part of the left pier (P2);
C6_ hinge at the upper part of the left pier (P4);
C7(a,b,c)_ 25%, 50% and 75% lacking from the lower part of the pier modelled with three-dimensional finite elements (P3);
C8(a,b,c)_ 25%, 50% and 75% lacking from the upper part of the pier modelled with three-dimensional finite elements (P3);
C9(a,b,c)_ lacking from the deck supporting area with: one third laterally, one third in the centre, two thirds from the cross section;
C10(a,b,c)_ lacking from the deck central area with: one third laterally, one third in the centre, two thirds from the cross section;
To these scenarios the initial situation of the entire structure is added – marked **C0**.

Table 1

Values of the longitudinal elasticity module.

Case name		***E*** [kN/m^2]	**Ratio from *E***	**% from *E***
Soft	C1a, C2a	30	*E*:1 000 000	10^{-4}
Medium 1	C1b, C2b	300	*E*:100 000	10^{-3}
Medium 2	**C1c, C2c**	**1 000**	***E*:30 000**	**3.3 10^{-3}**
Medium 3	**C1d, C2d**	**3 000**	***E*:10 000**	**0.01**
Medium 4	**C1e, C2e**	**30 000**	***E*:1000**	**0.1**
Medium 5	**C1f, C2f**	**100 000**	***E*:300**	**0.33**
Medium 6	C1g, C2g	300 000	*E*:100	1
Medium 7	C1h, C2h	3 000 000	*E*:10	10
Strong 1	C1i, C2i	7 500 000	*E*:3	25
Strong 2	C1j, C2j	15 000 000	*E*:2	50
Strong 3	C1k, C2k	22 500 000	*E*:(4/3)	75

Results. Comments

In this article only some results of the study will be presented, those obtained for the scenarios corresponding to initial damages at the piers level, based on their inherent dynamic response, Table 2.

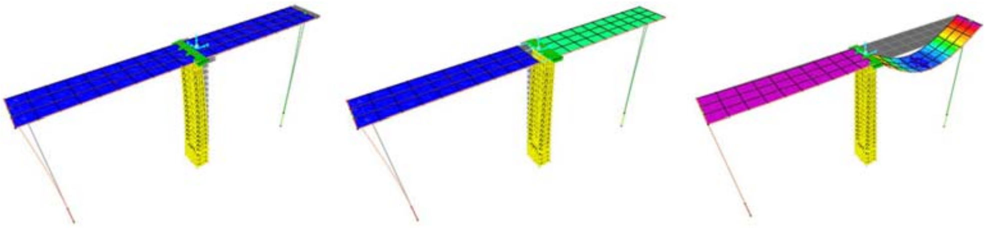
We consider that the mechanical values of their inherent dynamic response being an essential characteristic of the structure characterizing its general behaviour, may also give information on the way the progressive collapse phenomena may develop. The progressive collapse, although it starts from a minimum of the initially destroyed elements, appears due to the fact that the structure, on the whole, does not find the possibility to redistribute the appeared disturbance. From other studies carried out by the author [3,4] the conclusion was drawn that the efforts redistribution is made not necessarily to the elements adjacent to the removed one. Consequently, the structure behaviour shall be studied as a whole.

Table 2

The first three vibration modes for some of the scenarios referring to the initial damages in the piers.

		T1 [s] / Vector form	T2 [s] / Vector form	T3 [s] / Vector form
C0 – entire/whole		1.33933 longitudinally symmetrical	0.56577 Longitudinally non-symmetrical	0.46117 Bending the right span
C1 low hinge – three dimensional pier P3	C1e – medium 4	1.80836 longitudinally symmetrical	0.88945 transversal	0.61234 pier vertical movement, pulls the girders along
C3 – hinge, low pier P2		1.78186 longitudinally symmetrical	0.73864 Rotation round the pier P4 – after the vertical axis (z)	0.59249 longitudinally non-symmetrical
C4 – hinge, low pier P4		2.72690 transversal right span	1.51885 longitudinally symmetrical	0.57321 longitudinally non-symmetrical
C7a – 25% lack of low pier		1.43330 longitudinally symmetrical	0.58510 transversal	0.57323 longitudinally non-symmetrical
C7b – 50% lack of low pier		1.45953 longitudinally symmetrical	0.74848 transversal	0.57520 longitudinally non-symmetrical
C7c – 75% lack of low pier		1.46923 longitudinally symmetrical	0.84855 transversal	0.57626 longitudinal asymmetrical
C2 upper hinge – three-dimensional pier P3	C2d medium 3	1.49448 longitudinally symmetrical and cross head rotation as compared to transversal axis	0.74147 cross head: vertical movement draws the girders along	0.66491 cross head rotation, draws the girders along
C5 – hinge at upper pier P2		1.33933 equal with T1 from C0 longitudinal symmetric	0.56571 equal with T2 from C0 longitudinally non-symmetrical	0.46117 bending the right spans
C6 – hinge at upper pier P4		1.43581 rotation of right span - long axis (x)	1.33933 equal with T1 from C0 longitudinally symmetrical	0.56571 equal with T2 from C0 longitudinally non-asymmetrical
C8a – 25% lack of upper pier		1.33471 longitudinally symmetrical	0.56431 longitudinally non-symmetrical	0.53288 transversal
C8b – 50% lack of upper pier		1.33038 longitudinally symmetrical	0.56231 longitudinally non-symmetrical	0.53108 transversal
C8c – 75% lack of upper pier		1.32994 longitudinally symmetrical	0.56100 longitudinally non-symmetrical	0.54985 transversal and rotation round the longitudinal axis (x)

In Figure 7, the first three inherent vibration modes are presented for the situation when the structure is complete C0. Figure 8 presents the first three inherent vibration modes for the scenario with the hinges at the lower level of the pier modelled with three-dimensional finite elements C1e-medium 4 (E1e=*E*:1000).



(a) C0-mode 1 (b) C0-mode 2 (c) C0-mode 3

Fig. 7 – The first three modes of vibration for the case of the entire structure.

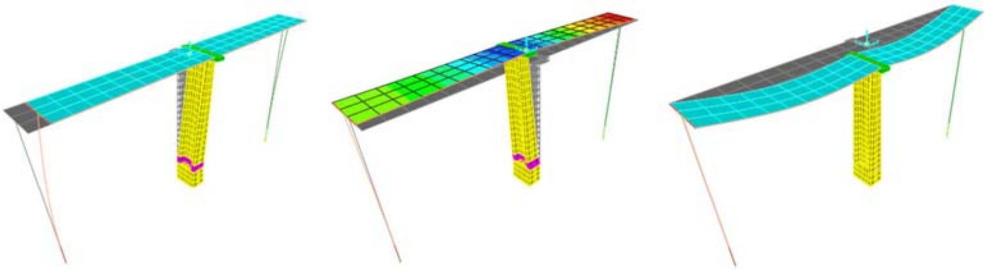


Fig. 8 – The first three modes of vibration for the scenario with a joint at the inferior level of the pile modelled with three-dimensional finite elements C1e-medium 4 (E1e=*E*:1000).

In a first stage, it was intended to choose a value of the longitudinal elasticity module of the material modelling the hinge at the three-dimensional pier, the value above or under which nothing interesting from the engineering point of view, happens.

The way the values of the inherent dynamic response were processed is under the form of the variation of the relative decrease of the values of the inherent periods calculated according to the formula (1):

$$D_{\text{relative}} = \frac{T_i - T_{i+1}}{T_i} = \frac{\Delta T}{T_i}\,. \tag{1}$$

In the diagrams in figure 9 there are presented the variation of the relative decrease (from one scenario to the other) of the inherent periods calculated according to the formula (1)

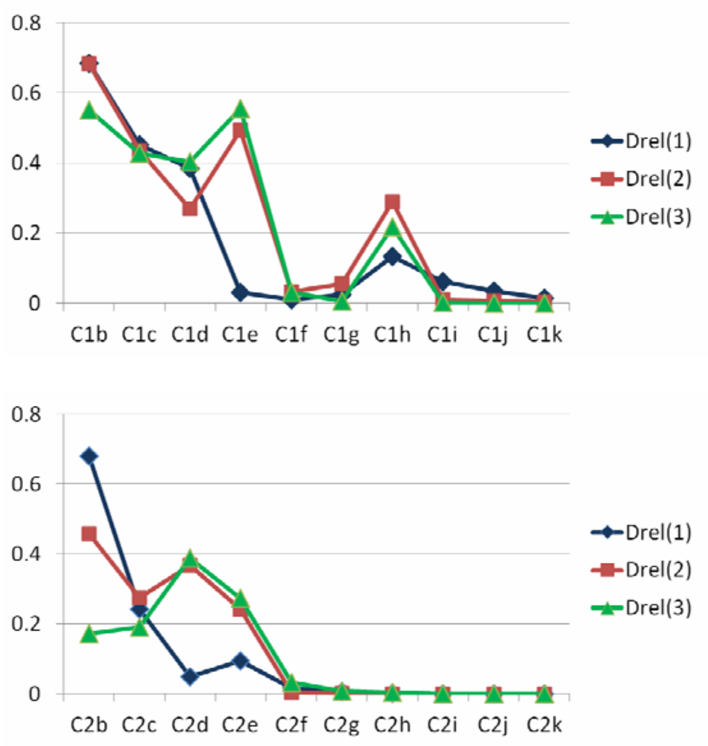


Fig. 9 – Variation of the relative decreases (from one scenario to the next) of the eigenvalues calculated with formula (1)

The variation of the relative decrease $D_{\mathbf{relative}}$ of the values of the fundamental period forms a relatively continuous curve. The variation of the relative decrease D_{relative} of the values of the modes 2 and 3 periods has not an even distribution.

For the case where the *hinge is at the lower part:*

- in the scenario C1e (E1e=E:1000) the structure response is the most different related to the relative decrease of the inherent vibration periods.
- the scenarios where the structure has the most appropriate response in the three inherent modes are C1c (E1c=E:30000) and C1f (E1f=E:300).

For the case where the *hinge is at the upper part*:

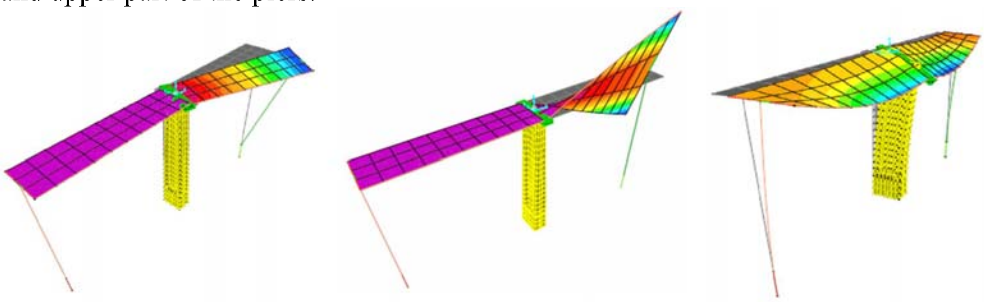
- the scenario with the most differences is C2d (E2d=E:10000).
- The first scenario where the values of the decreases for the three inherent periods are the closest is C2f (E2f=E:300).

* Between the cases C1c (C2c) and C1f (C2f) there is an area where the values D_{relative} are different and, then, I propose the elasticity module to be chosen from this interval of values – from E:30.000 to E:300. At a first sight, the interval seems to be large but the values may be chosen in correlation with the extent of the area of the initial damage.

* The inherent vibration forms up to the scenarios C1e and C2d come from the movements generated by a rigid body. The above mentioned scenarios are the first where the fundamental forms comes back to the fundamental form of the scenario C0-undamaged structure.

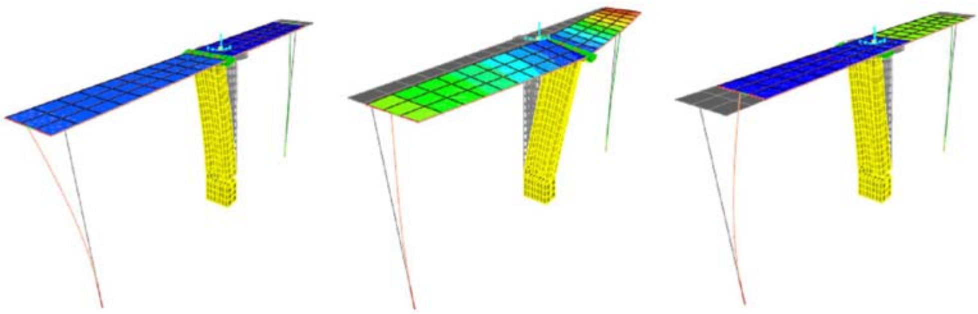
* A first conclusion is that for the longitudinal elasticity module cannot be chosen only one value both for the scenario with a hinge at the lower part of the pier and for the scenario with a hinge at the upper part of the pier. To go on with the study the two scenarios were chosen where the response is the most different, namely: for the hinge at the lower part of the pier the scenario C1e (E1e=E:1000) and for the hinge at the upper part of the pier scenario C2d (E2d=E:10000).

Figure 10 presents some forms of the specific vectors having a different aspect. In Figure 11 there are presented the first three specific vibration modes for the scenario C7b_50% lack at the lower part of the pier modelled with three-dimensional finite elements. In Figure 12 are the graphical representation of the eigenvalues of the first three specific vibration modes for the cases: a) hinge at the low part of the piers; b) hinge at the upper part of the piers; c) hinge both at the low and upper part of the piers.



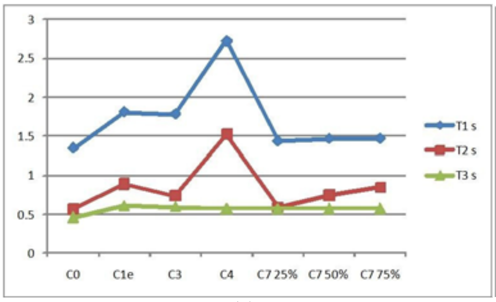
fundamental mode – scenario C4 fundamental mode – scenario C6 mode three – scenario C8c

Fig. 10 – Eigenvectors for various scenarios: a) fundamental mode – scenario C4; b) fundamental mode – scenario C6; c) mode three – scenario C8c

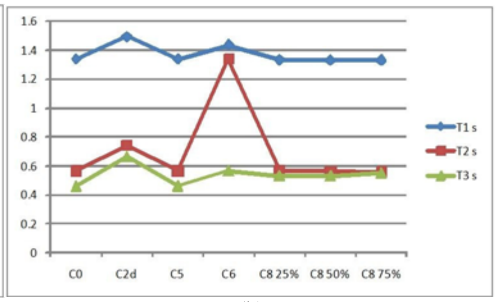


C7b – mode 1 C7b – mode 2 C7b – mode 3

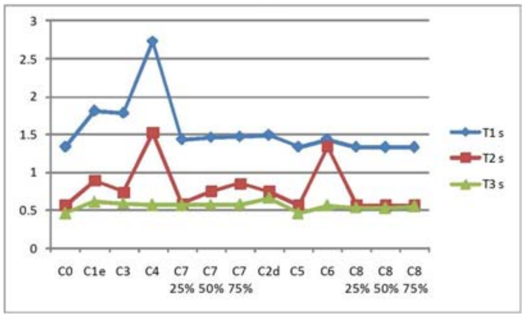
Fig. 11 – The first three modes of vibration for the scenario C7b_lacks over 50% of the pile modelled with three-dimensional finite elements.



(a)



(b)



(c)

Fig. 12 – Graphical representation of the eigenvalues of the first three modes of vibration for the following scenarios: a) joints at the inferior level of piers; b) joints at the superior level of piers; c) joints at both inferior and superior level.

Based on Table 2 and Figure 10, the following comments can be done regarding the specific dynamic response (values of the specific periods and forms):

* The inherent period of the fundamental module for the structure when it is complete show a flexible behaviour. Consequently, I considered as necessary, at least for the study, the modules two and three as well.

For the fundamental mode:

* The periods of the fundamental module has higher values for the scenarios with a hinge at the low part of the piers having a peak for the scenario C4-hinge at the low part of pier P4 (the highest pier in the structure). The ratio of the fundamental periods C4/C0 is 2.04.

* Speaking about the form of the vibration vector, it varies only in the scenarios referring to pier P4. The change is only for the span linked to the pier P4 and does not involve the whole structure.

* Scenarios C4 and C6 corresponding to pier P4 modifies the fundamental mode, but the mode 2 and mode 3 are identical with the fundamental mode and the mode 2 of the undamaged structure – it appears like a sort of coming back to the behaviour of the undamaged structure after the initial damage had consumed its effect.

For mode 2:

* The diagram for the periods of the mode two show higher variations. There are two peaks corresponding to the scenarios for pier P4. The maximum ratio, which is also for the periods C4/C0, this time is 1.71. In the same time, one can see that the period values of the scenarios C1e and C7 are practically equal.

* Although the value of the specific period of mode two of the scenario C4 is the highest, I do not consider this scenario as the most dangerous, but that where the form of the specific vector changes into vibration in the transversal direction on the bridge (C1e, C7a, C7b, C7c).

* Scenario C5-hinge at the upper part of the pier P2 (where the girders have hinged bearings) does not change at all the dynamic response of the structure, being identical in point of periods values and form of the specific vectors with the response of the undamaged structure for all the three inherent modes.

For mode 3:

* The values of the specific periods are relatively constant. The scenarios having the highest periods are C2d (the ratio of periods C2d/C0 is 1.44) and C1e.

In conclusion, one can say the structure is robust as compared to those scenarios leading to amplifying the longitudinal movement on the bridge. In the same time, the scenarios that have as effect the appearance of the movement on the bridge transversal direction may lead to collapse

4. CONCLUSIONS

The structure which is the object of this study is a road concrete viaduct. The initial damages – situated in the piers or superstructure – form 38 scenarios to which the situation of the undamaged bridge is added. In this article are presented the results and comments referring to the scenarios for the damages, at the piers level, carried out based on the specific dynamic response of the structure. The piers were modelled with one-dimensional and three-dimensional finite elements. To use the finite elements to model the same types of structural elements leads to the necessity to model the damage differently as well even if it is of the same type.

Thus, to model that type of damage when a section of the pier modelled with three-dimensional finite elements changes into a hinge, a new material was introduced and defined by its elasticity module and specific weight.

Due to the structure different response when hinge type degradations appear at the low or upper level of the pier it was necessary to define two different elasticity modules for the two positions of the damaged section.

The scenarios that lead to a dynamic response with many different elements as compared to the answer of the undamaged structure are those referring to the change of a section at the low part of a pier into a hinge.

In case there are more scenarios a translation of the dynamic response appears, namely the mode two of the structure with the respective damage looks like the fundamental mode of the undamaged structure and the three mode of the damaged structure looks like the mode two of the undamaged structure. The fundamental mode of the damaged structure consumes the damage influence and the structure has resources to regain its initial behaviour in the superior modes. The best example is that of the scenarios C4 and C6 corresponding to hinge type damages at pier P4 – the highest in the structure.

From the carried out study I can draw the conclusion that the structure has resources to preserve its general initial behaviour for the damages amplifying the movement in the longitudinal direction.

In the same time, scenarios were also rendered evident where a new form of the specific vibration vector appears, that is in the transversal direction on the bridge, drawing along the whole structure and that is able to bring about its collapse.

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